ARIZONA DEPARTMENT OF TRANSPORTATION

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ALTERNATE CONCRETE ACCEPTANCE CRITERIA

State of the Art

Final Report

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PREFACE

This report was sponsored by the Arizona Transportation Research Center through the man-year faculty agreement with the Center for Advanced Research in Transportation. The objective of the study is to review present practice used in evaluating and accepting concrete and to recommend any needed changes in current ADOT specifications.

TABLE OF CONTENTS

I. Introduction

Evaluating the quality of portland cement concrete and developing a rational acceptance criteria related to material test specimens has been a matter of study and discussion among engineers, architects, and concrete technologies for the past 50 years. As time passed and concrete was used in more sophisticated designs, structural integrity and safety became more and more important. Many opinions and philosophies exist in the continuing efforts to develop sound criteria for acceptance or rejection of in-place concrete. This is based essentially on the results of field fabricated concrete test specimens which conceptually indicate the quality of the concrete in the structure.

The need for a reliable, repeatable, simple, and rapidly performed field test for such acceptance has remained centered around one item; the compressive strength of the concrete as measured by the destructive testing of job-fabricated test cylinders. The results of these tests are then compared to the design strength required by the project.

It is often assumed that any indication that the concrete test cylinder fails to either meet or exceed the design strength implies a corresponding lack in integrity in the portion of the concrete structure represented by the test specimens. On this point alone, debate becomes heated.

While there is no debate over the purpose and need for such testing to assure needed structural safety, there are rational questions over what constitutes an acceptable concrete strength test. The major factor in such debate centers around the heterogeneous nature of concrete. When concrete is produced in a plant, it is subject to variability involving the ingredients that make up the concrete mix; i.e. the cement, aggregates, water and admixtures. In addition, the mixing procedures, transportation, placement and curing practices are among many additional factors that can add to variability in the final product. If we then add variabilities in testing procedures, it becomes readily apparent why there are variabilities in the concrete, and in the test specimens the concrete is represented by.

It is inherently necessary to establish acceptance criteria that recognize these variabilities. Furthermore, this must be done without compromising the safety and integrity of the structure itself.

II. An Overview of Various State Highway Department Approaches to Evaluating Concrete

The Arizona Department of Transportation's current acceptance criteria requires that the average of two test cylinders fabricated at the time of concrete placement attain a minimum compressive strength of 95 percent of the 28-day design strength. If this criteria is not met, the contractor is permitted to take cores of the suspected concrete within 42 days of the placement. If the average of three test cores, tested in a wet condition, equals or exceeds the required design strength, the concrete is accepted without penalty.

This criteria is similar to the criteria used by a number of other state highway agencies. However, the detailed requirements for portland cement concrete varies considerably between the state highway agencies in both mix design and in acceptance criteria related to low compressive strength cylinder breaks.

This wide variety of specification and departmental procedures is portrayed in Table 1. It is important to recognize the similarities and disparities of the data in this table. Only then does the complex nature of agency criteria for concrete and its minimal acceptance criteria become fully apparent. Several items of note in this regard:

1. The printed specification of all the state highway agencies (SHA), with the exception of Vermont, require a minimum cement content in their concrete mix design.
2. All SHA limit the maximum water-cement ratio, with the exception of Arizona, California, Michigan, and South Dakota.
3. Compressive strength targets or ranges for various classes of concrete are defined in approximately 60 percent of the

TABLE 1

State	Classes of Concrete	Range of Min. Cement Contents	Range of Max. Water-Cement Ratios	Compr. Strength Range	Misc. Notes re: Acceptance and Evaluation Criteria
Alaska	4	470 - 658	0.51 - 0.58	(1)	
Arizona	3	517 - 752	---	2500 - 4000	Min. 95% 28 day compr. str. or cores at 42 days to meet f'c-tested wet. Penalty clause
Calif.	4	564 - 800	---	(1)	Min. 95% moving ave.; 85% all ind. tests.
Colo.	11	565 - 700	0.44 - 0.53	(1) (1)	Uses special formula for acceptance and price adjustment.
Del.	4	423 - 705	0.40 - 0.60	2000 - 4500	Requires mix with design strength plus 300 psi std. deviation.
Fla.	4	564 - 658	0.41 - 0.55	2500 - 5000	Uses core tests for low strengths plus penalty clause to compensate state for loss of durability. (5)
Geo.	6	470 - 675	0.44 - 0.66	2200 - 5000	Designs mixes for f'c plus 2 std. deviations. Uses ACI criteria for acceptance of low breaks.
Idaho	8	380 - 700	0.44 - 0.60	(1)	Min. 95% using moving averages. Price adjustments to 85%. Cores tested dry to check low strengths (6)
Ill.	3	(2)	(2)	(3)	
Ind.	3	470 - 611	0.49 - 0.62	(1)	Only mentions beam breaks (flexural) to check form removal.
Iowa	3	583 - 710	0.41 - 0.43	(1)	
Kansas	5	526 - 696	0.44 - 0.58	(1)	
Kentucky	6	451 - 733	0.44 - 0.66	2500 - 5000	Low cylinders reviewed per KM 64-314-86. Requires in-place investigation after 500 psi under design, with cores. (9)
La.	10	376 - 658	0.48 - 0.71	2000 - 6000	
Maine	5	517 - 635	0.45 - 0.53	3000 - 4000	Uses statistical evaluation per ACI 214. Accepts 1 out of 10 low breaks in 3000 & 4000 psi mixes.
Mich.	9	450 - 658	(4)	3000 - 4500	Has special departmental review process not stated in specs.
Missouri	6	564 - 752	4.5 - 6.0	(1)	Only mentions compressive str. required for form and falsework removal.
Mont.	8	517 - 752	5.0 - 6.0	2400 - 5000	Requires mix design + 400 psi std. deviation and invokes penalty clause at 300, 400, & 500 psi under to 70% pay.
Nebr.	11	564 - 822	0.44 - 0.59	1500 - 3500	

Nev.	8	470 - 752	0.51 - 0.60	2500 - 3000+(1)	Acceptance by moving averages plus penalty deduct for up to 15% under design strength. (7)
NH	5	489 - 658	0.44 - 0.56	(1)	
NJ	7	517 - 705	0.44 - 0.62	4500 - 6500	Allows up to 10% material below class strength. Beyond that limit uses a quality index formula and pay adjustment (8)
N. Mex.	8	470 - 658	0.46 - 0.53	2200 - 4000	No formal method; uses engineering judgment and no pay penalty for acceptable low strengths on structures
N.Y.	10	517 - 727	0.38 - 0.46	(1)	Allows a 10% variation below design strength of 7-day cylinders.
No. Car.	6	508 - 639	0.38 - 0.49	2500 - 4500	Allows up to 10% below strength at 7 days only.
No. Dak.	3	517 - 611	0.44 - 0.55	(1)	
Ohio	3	470 - 715	0.44 - 0.55	4000 - 4500	Uses core tests for low strengths at 28-days. Evaluates with a quality index and has penalty clause to 0 pay.
Okla.	6	395 - 752	0.42 - 0.58	(1)	Compressive or flexural strength requirements only for form removal or early pavement service.
Oregon	10	470 - 660	0.40 - 0.79	(1)	Uses statistical evaluation and quality level analysis and pay penalty.
Penn.	6	395 - 846	0.40 - 0.66	2000 - 4500	7-day strength cyls. used for acceptance. 28-day results show potential of design mix.
R.I.	7	423 - 705	0.42 - 0.66	2000 - 5000	
So. Car.	5	494 - 705	0.42 - 0.66	2500 - 5000	Mix designs must result in 28-day strengths 25% above design strengths, or change mix.
So. Dak.	3	500 - 600	---	3000 - 4000	Uses engineering judgment without written procedure, on a job by job basis.
Tenn.	2	620 - 658	0.44 - 0.53	(1)	Contractor allowed to core for low strengths. State will accept low strength if judgment indicates durability O.K. (11)
Tex.	10	250 - 900	0.37 - 0.98	1500 - 5500	Statistiacal evaluation and follows ACI 318 with cores and 90 day.
Utah	4	376 - 564	0.94 - 0.71	2000 - 3650	Uses moving averages (1 in 100 statistical evaluation) Pay adjustment to 80% for 400 psi under minimum. (12)
Vt.	6	---	0.40 - 0.58	2500 - 4000	
Virg.	6	423 - 635	0.45 - 0.71	1500 - 5000	Specs. require mix adjustment for low strengths, but no acceptance criteria or pay penalty.

Wash.	3	470 - 650	0.42 - 0.68	(1)	Coring and penalty clauses used.
W. Va.	5	376 - 682	0.44 - 0.62	2000 - 4500	Acceptance by statistical evaluation and pay penalty by formula.
Wisc.	5	400 - 823	0.49 - 0.53	(1)	
Wyo.	5	517 - 635	0.44 - 0.55	2750 - 5000	

(1) As indicated on project plans or supplemental specifications.

(2) Illinois requires 605 lbs. cement/cy and a 0.48 w/c ratio for bridge deck concrete only.

(3) Illinois uses compressive strengths as indicated on project plans except for bridge decks which require 4000 psi in 14 days.

(4) Maximum w/c ratio of 0.44 required for bridge deck construction only.

(5) Contractor develops a project/mix design relationship between cores and cylinders after 42 days.

(6) Three cores converted to 28 day strength using Idaho T-89. (Any age to 90 days)

(7) Allows cores for low strengths; corrected to max. age 90 days.

(8) Cores acceptable for retest, per AASHTO T24

(9) Allows to 500 psi low without special investigation--and without penalty clause.

(10) 14-day compressive strength tests used for acceptance or special evaluation.

(11) Cores not meeting design strengths will result in pay reduction by formula using ratio of 28-day strength/specified strength.

(12) Cores taken at 35 days or earlier.

specifications; apparently the others identify required strengths on plans and supplemental specifications.

4. Specific acceptance and evaluation procedures were shown in less than 40 percent of the state specifications reviewed. Some states, exemplified by New Mexico, Texas and Washington have no documented evaluation and/or acceptance guide or procedure. They apparently use engineering judgment on an individual project basis, without a rigid guideline.
5. Levels of acceptance of low cylinder breaks, among those states with specific procedures, included 85, 90 and 95 percent of design strength.
6. Statistical evaluation and/or methods involving standard deviations and moving averages or strength tests were specifically detailed by approximately 20 percent of the SHA.
7. The specific use of test cores to "override" low strength cylinder test results was mentioned in 11 state specifications. Both wet and dry tests are used, with the difference usually related to the degree of moisture available to the structural element during its service life.

III. A Comparative Review of Other Public Agency and Private Sector Evaluation Techniques

In view of the variability in specifications between the various state highway agencies, recommendations and experiences of other governmental agencies and private organizations were reviewed. The following were selected in this regard:

Federal Highway Administration (FHWA)

US Army Corps of Engineers (COE)

US Bureau of Reclamation (USBR)

Maricopa Association of Governments (MAG)

National Ready Mix Concrete Association (NRMCA)

American Concrete Institute (ACI)

Portland Cement Association (PCA)

American Concrete Paving Association (ACPA)

The Federal Highway Administration, as detailed in their Standard Specifications for Roads and Bridges on Federal Highway Projects (2), uses a set of four (4) concrete test cylinders as a basis for accepting or rejecting any particular lot of concrete. The first two are tested at 28 days with the remaining two tested for verification at a later date. Any concrete represented by cylinders having a compressive strength of less than the required f'c is evaluated statistically using a 'Quality Level Analysis.' This utilizes standard deviations of the mixes involved and results in pay factor reductions down to 75 percent. Concrete with a quality lower than the minimum is not necessarily taken out, but is judged from an engineering basis, and could result in acceptance without pay under certain circumstances. This analysis also results in the contractor receiving up to 5 percent reward pay for concrete that exceeds required levels of quality. Coring is not addressed in the document, but apparently can be used by the contractor to prove a case.

The U.S. Army Corps of Engineers has a somewhat less structured method of acceptance, and conversations with engineers in both their Phoenix office and the Sacramento headquarters indicate their commitment to performance specifications. The usual procedure at the local Phoenix office, if the 28-day cylinder strengths are not met, is to wet cure the structure and take cores, usually at 56 days. According to the local COE spokesman all concrete tested with this method has been accepted.

The Sacramento staff is approaching the problem on a more sophisticated basis. They are trying to follow the acceptance procedures of Chapter 4 of the ACI Building Code (318-83) (5). Furthermore, they do not use a penalty clause or reduction for marginal concrete. If the concrete meets the statistical evaluation procedure, or is accepted from the coring procedure, full payment is given.

The U.S. Bureau of Reclamation has criteria used by area and project engineers which requires the strength of 80 percent of all test specimens for a given lot of concrete be equal to or greater than the design strength, using a running average of ten tests (3). They also assign levels of coefficients of variation for each class of concrete from 2,000 to 6,000 psi concrete. This procedure has apparently worked quite well for them in the past, and they indicate that they could not remember any concrete on the Central Arizona Project canal system not meeting the specifications of requiring removal of inferior concrete for reasons of low strengths. This may be changing now, however, for they are now building the distribution network of laterals and ditches, and these use ready-mix concrete suppliers rather than the central plants used for the main canal. They are now having occasional failures, but try to minimize this by allowing cores to be taken up to 90 days and accepting if f'c is met by that time.

The National Ready Mix Concrete Association does not have any specific documents relating to acceptance and evaluation, but enthusiastically supports the trend in which many highway departments and other agencies are going in adopting the provisions of Chapter 4 of the ACI Building Code.

NRMCA has also indicated its favorable impression with the results of West Virginia's program which has been developed over a 15 year period of trial and improvement. Basically, it requires the overdesign of all concrete mixes based on the efficiency and track record of any given concrete plant or source. It is felt this has reduced the number of low breaks to such a small number that the problem of acceptance becomes a moot point.

The Maricopa Association of Governments' guide specifications for public works construction (4) recommends accepting concrete when at least 95 percent of the required 28-day compressive strength is obtained. All concrete which fails to meet this criteria must be removed. However, the contractor may use core tests to prove the in-place concrete meets this minimum. There is an adjustment in contract unit price for the strength deficiency between 95 and 100 percent of f'c. This penalty ranges from 0 to 20 percent.

Lastly, the recommendations of the American Concrete Institute and the Portland Cement Association are of particular note in this review of acceptance and evaluation techniques. The ACI procedures are contained in the Building Code Requirements for Reinforced Concrete (ACI 318-83); a document fully embraced and supported by the P.C.A.

Chapter 4 of this document is particularly important in this regard, and is contained in its entirety in Appendix A of this report. Of all the various methods available and used by the various agencies, this procedure presents perhaps the most efficient method. It uses a three-step approach centered on statistical evaluation. Specifically, this ACI procedure states that the strength level of any individual class of concrete shall be considered acceptable if:

a. The average of all sets of three consecutive tests be equal to or exceed the design strength (f'c) and,

b. That no strength test (defined as the average of two cylinders) falls below f'c by more than 500 psi.

If these two criteria are not satisfied, further investigation is required:

a. If any test is 500 psi less than f'c, steps shall be taken to assure load-carrying capacity of the structure is not jeopardized.

b. If the likelihood of low strength concrete is confirmed and computations indicate that load carrying capacity may have been significantly reduced, cores may be required.

c. If cores are taken, 3 cores shall be taken for each strength test that was 500 psi less than f'c.

d. If the structure will be dry under service conditions, cores shall be air dried at 60° to 80°F with relative humidity less than 60 percent for seven days and then tested dry.

   If the concrete in the structure will be more than superficially wet under service conditions, cores shall be immersed in water for at least 48 hours, and tested wet.

e. Concrete from the area represented by the core shall be considered structurally adequate if the average of three cores is equal to or at least 85 percent of f'c and if no single core is less than 75 percent f'c. Erratic core strengths may be retested.

f. If all these fail, load the structure (per Chapter 20 of ACI 318-83) and test for allowable deflections.

IV. Implications of Adopting End-result Specifications Using Statistical Evaluation

After review of all the aforementioned data, it would seen logical and progressive to consider adopting a specification which would:

a. Use statistical evaluations of compressive strength tests per ACI 214-R83 (6)and,

b. Allow the acceptance of compressive strength tests to as much as 500 psi under the design strength (f'c) per ACI 318-R83

This would probably assume that the contractor be allowed full discretion in designing the concrete mix. Minimum cement contents and maximum water/cement ratios would not be imposed. The concrete's compressive strength then becomes the one and only determining factor for acceptance. It then becomes necessary to explore both literature and experience to equate compressive strength against another particularly important property of concrete, its durability.

A material's durability is usually defined as its ability to withstand wear and tear or decay and be long-lasting. A more useful definition for our purpose is perhaps to relate durability to service life. The durability of any building material, including concrete, is an important property but is not a well-defined, directly measurable quantity. As most often used, durability is a term which describes human opinion as to performance under a range of conditions to which it is to be exposed.

Durability is unquestionably important and there have been many studies comparing strength and durability (7,13,14,15). More importantly, many recent studies show that it is no longer valid to assume that strength and durability increase together predictably. It has been assumed that increases in compressive strength result in corresponding increases in durability. There are now several studies and authored papers (9,10,16) which indicate compressive strengths have dramatically increased over the past decade or so, but durability has apparently not kept pace with these improvements in strength values.

Editorials and warnings began to be published with regard to this breakdown in the traditionally expected strength-durability relationships. In 1981 the Portland Cement Association indicated apparent improvements in many portland cements which resulted in somewhat higher strength concrete for given quantities of cement. The manufacturers' tendencies to grind their product finer was a part of this (7). G. Fagerlund, a renowned Swedish cement technologist, indicated that from studies (8) he conducted that the general level of durability of concrete in 1981 was less than it was 10 or 15 years earlier. He reasoned that this was related to water-cement ratios having increased for given strength levels, but their corresponding durabilities had actually decreased. His actual figures indicated that for a given strength, water-cement ratios increased 10 percent, while accompanied by a five-fold increase in permeability in the concrete and a corresponding reduction in durability.

A year later, the PCA once again stated its continuing concern about the durability of concrete structures exposed to the natural environment (9). There was considerable evidence accumulating which showed increases in strength taking place as water-cement ratios increase--and permeability of the paste decreased. The resulting loss in durability reinforced the point that an equality sign cannot be placed between strength and durability. The use of compressive strength alone introduces a risk of excessive permeability of the concrete and a corresponding loss in durability. Many reports and papers support this hypothesis (13,14,15). In an unpublished article by Rasheedduzzafar, Dakhil and Al-Gahtani in 1982 (10) the following is noted:

a. Environmental factors such as corrosion of reinforcement, sulfate attack, aggressive chemicals, weathering, and cracking due to thermal and moisture gradients affect concrete performance so greatly that properties of concrete other than strength become very significant.

b. Disintegration due to one factor often accelerates another form of attack and total deterioration is cumulative.

c. One must not fail to recognize the preeminent role of permeability in governing concrete's durability performance in aggressive environments. This is exemplified by rebar corrosion and sulfate attack, both of which are extremely permeability oriented.

d. The preeminence of permeability in governing concrete and durability characteristics implies laying specific provisions, along with appropriate strength grades, to produce low permeability concrete.

e. Using modern techniques of quality control, it is possible to achieve adequate compressive strengths with rather low cement contents. This results in a lack of paste density and a corresponding increase in permeability. Sulphate attack and weathering resistance are especially affected by this.

f. Corrosion of reinforcing bars is also enhanced with these higher water-cement factors. A lack of plastic consistence fails to provide alkaline protection usually provided by a high quality, evenly coated cement paste on the steel bars.

It would seem prudent, then, to acknowledge that today's technology allows the design of higher strengths with less cement or higher water-cement ratios. The result, however, is more permeable pastes in the concrete system. It is then necessary to use water-cement ratios and/or minimum cement contents as a control mechanism. This is unusable, however, if we wish to use any form of end-result specification. To do that we must use compressive strength tests, and levels of acceptable strength must be chosen that will assume higher quality of the paste material in a concrete mix. We must therefore choose compressive strengths for job control and acceptance well above that required by the structural design needs of the project. The addition of 500 psi to 28-day compressive strengths is recommended by Fagerlund (8) and endorsed by the Portland Cement Association (7,9).

If we are to use these higher compressive strengths for acceptance and evaluation of concrete, we must next explore what methods to use.

V. The Argument for the Statistical Acceptance of Compressive Strength Cylinders

The role of a compressive strength cylinder is well stated in ACI 214-R83 (6). These test specimens indicate the potential rather than the actual strength of the concrete in a structure. To be meaningful, conclusions on strength must be derived from a pattern of tests from which the characteristics of the concrete can be estimated with reasonable accuracy.

The relationship between test cylinders and concrete in a structure has been studied seriously beginning with H. H. Edwards 1929 work at Scripps College (11). The Swedish Cement and Concrete Research Institute updated European practice and thinking in 1964 (16). D. L. Bloem followed that with a call to consider separating our concepts of strength into two distinct categories: design strength for calculation of load-carrying capacity, and control strength for measuring proper quality and uniformity of concrete used in the work (17).

Accepting or rejecting concrete on the basis of one test cylinder or the average of two or three cylinders has been adequately questioned. In its place a pattern of tests, or statistical evaluation, finds growing acceptance. Of the 14 highway agencies that use this or a related approach, New Jersey's statistical based specifications for portland cement concrete is particularly noteworthy (18, 19). They have successfully used statistical acceptance procedures for several years.

This NJDOT specification is of the end-result type with the contractor responsible for most of the control of both product and work, while the highway agency retains responsibility for final acceptance. That department still holds to some remnants of method specifications, i.e., minimum cement factors and maximum water-cement ratios. A pay adjustment factor is calculated from an equation which recognizes an acceptable percent of defect below specification limits. Experience has shown that small amounts of percent defective can be tolerated without detracting from the serviceability of a structure, etc. ACI 214 discusses the philosophy of this quite well (6).

The big departure and new innovation by NJDOT involves the inclusion of an awards bonus for quality that is substantially in excess of the required level. This is limited to 102 percent of pay item. New Jersey argues that whenever an acceptance procedure is based on the percent defective parameter and a quality level other than zero-percent defective is stated (or implied) to be acceptable, pay factors of 100 percent are required if the specification is to perform fairly (18). This type of positive incentive provision is supported by the Federal Highway Administration.

New Jersey is apparently pleased with results to date and continues to assess the new procedures as they affect additional contracts. The other S.H.A.'s that use a statistical approach also find the procedures workable. Although the reward bonus is not adopted by other states, penalty clauses and penalty pay factors are invoked by at least 11 other S.H.A.'s

Statistical evaluation is also endorsed by the construction industry. M. Lee Powell, President of the Ballanger Group and speaking for that industry, has stated their concern that highway construction technology is advancing faster than highway specifications (20). He argues that absolute conformity cannot be achieved at any reasonably acceptable cost and the principle of reasonably close conformity must take its place. The statistical concept is a valuable tool in defining just what reasonably close conformity is. It is a rational method of setting numerical limits so that the average quality of all construction will be satisfactory. In this way the specification limits fully account for the minimal variations in test results that occur from random curves. The quality control program then becomes designed to detect, locate, and correct serious deviations resulting from assignable causes.

VI: Conclusions and Recommendations

The current Arizona Department of Transportation criteria for acceptance of concrete, although similar in some respects to a number of other state highway agencies, is in need of updating and improvement.

ADOT is apparently one of only four state highway agencies that does not limit maximum water to cement ratios in their concrete specifications. Without this limiting factor a contractor can use sophisticated admixture-based mix designs and have little trouble meeting compressive strength requirements. The resulting concrete often has water-cement ratios higher than that necessary to achieve needed levels of durability. To resolve this issue, ADOT can either use a method specification, including maximum water-cement ratios, to insure both strength and durability, or use an end-result specification. The latter should certainly be adopted. In addition, durability should be an important factor in any such a specification. Durability is hard to quantify at time of construction, however, and compressive strength remains the only logical criteria to use for acceptance. An increase in compressive strength levels for all classes of concrete would allow this to be achieved while still guaranteeing needed durability. An increase of 500 psi over the 28-day design strength has been suggested in the literature. Whether or not this is an exact value for use in Arizona's environment should be researched.

Considering the variability associated with concrete properties, as well as with testing procedures, statistically based acceptance of compressive strength results must be considered for adoption. Statistical evaluations are widely used by many agencies for quality control purposes, and particularly for portland cement concrete. Chapter 4 of the American

Concrete Institute's Building Code Requirements for Reinforced Concrete (ACI 318-83), (included as Appendix A) represents the state-of-the-art in the quality control of concrete. This document recognizes statistical evaluations and is being adopted on an ever increasing basis by industry and agencies alike. We recommend that it be adopted for use in any revisions ADOT makes in their present specifications regarding concrete accceptance and evaluations. This can be adopted as a complete document or in carefully selected parts. In addition, any such revisions using this standard should also include the increases in compressive strengths (for acceptance), as mentioned above.

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APPENDIX A

American Concrete Institute

Building Code Requirements for

Reinforced Concrete

ACI 318-83

Chapter 4

PART 3 – CONSTRUCTION REQUIREMENTS

CHAPTER 4 – CONCRETE QUALITY

4.0 – Notation

f'_c = specified compressive strength of concrete, psi
f_{ct} = average splitting tensile strength of lightweight aggregate concrete, psi
f'_{cr} = required average compressive strength of concrete used as the basis for selection of concrete proportions, psi
s = standard deviation, psi

4.1 – General

4.1.1 – Concrete shall be proportioned to provide an average compressive strength as prescribed in Section 4.3.2. Concrete shall be produced to minimize frequency of strengths below f'_c as precribed in Section 4.7.2.3.

4.1.2 – Requirements for f'_c shall be based on tests of cylinders made and tested as prescribed in Section 4.7.2.

4.1.3 – Unless otherwise specified, f'_c shall be based on 28-day tests. If other than 28 days, test age for f'_c shall be as indicated in design drawings or specifications.

4.1.4 – Where design criteria in Sections 9.5.2.3, 11.2 and 12.2.3.3 provide for use of a splitting tensile strength value of concrete, laboratory tests shall be made in accordance with "Specification for Lightweight Aggregates for Structural Concrete" (ASTM C 330) to establish value of f_{ct} corresponding to specified value of f'_c.

4.1.5 – Splitting tensile strength tests shall not be used as a basis for field acceptance of concrete.

4.2 – Selection of concrete proportions

4.2.1 – Proportions of materials for concrete shall be established to provide:

(a) Workability and consistency to permit concrete to be worked readily into forms and around reinforcement under conditions of placement to be employed, without segregation or excessive bleeding.

(b) Resistance to special exposures as required by Section 4.5.

(c) Conformance with strength test requirements of Section 4.7.

4.2.2 – Where different materials are to be used for different portions of proposed work, each combination shall be evaluated.

4.2.3 – Concrete proportions, including water-cement ratio, shall be established on the basis of field experience and/or trial mixtures with materials to be employed (Section 4.3), except as permitted in Section 4.4 or required by Section 4.5.

4.3 – Proportioning on the basis of field experience and/or trial mixtures

4.3.1 – Standard deviation

4.3.1.1 – Where a concrete production facility has test records, a standard deviation shall be established. Test records from which a standard deviation is calculated:

(a) Shall represent materials, quality control procedures, and conditions similar to those expected and changes in materials and proportions within the test records shall not have been more restricted than those for proposed work.

(b) Shall represent concrete produced to meet a specified strength or strengths f'_c within 1000 psi of that specified for proposed work.

(c) Shall consist of at least 30 consecutive tests or two groups of consecutive tests totaling at least 30 tests as defined in Section 4.7.1.4, except as provided in Section 4.3.1.2.

4.3.1.2 – Where a concrete production facility does not have test records meeting requirements of Section 4.3.1.1, but does have a record based on 15 to 29 consecutive tests, a standard deviation may be established as the product of the calculated standard deviation and modification factor of Table 4.3.1.2. To

TABLE 4.3.1.2 – MODIFICATION FACTOR FOR STANDARD DEVIATION WHEN LESS THAN 30 TESTS ARE AVAILABLE

No. of tests*	Modification factor for standard deviation†
less than 15	Use Table 4.3.2.2
15	1.16
20	1.08
25	1.03
30 or more	1.00

*Interpolate for intermediate numbers of tests.

†Modified standard deviation to be used to determine required average strength f'_{cr} from Section 4.3.2.1.

be acceptable, test record must meet requirements (a) and (b) of Section 4.3.1.1, and represent only a single record of consecutive tests that span a period of not less than 45 calendar days.

4.3.2 – Required average strength

4.3.2.1 – Required average compressive strength f'_{cr} used as the basis for selection of concrete proportions shall be the larger of Eq. (4-1) or (4-2) using a standard deviation calculated in accordance with Section 4.3.1.1 or Section 4.3.1.2.

$$f'_{cr} = f'_c + 1.34s \qquad (4\text{-}1)$$

or

$$f'_{cr} = f'_c + 2.33s - 500 \qquad (4\text{-}2)$$

4.3.2.2 – When a concrete production facility does not have field strength test records for calculation of standard deviation meeting requirements of Section 4.3.1.1 or Section 4.3.1.2, required average strength f'_{cr} shall be determined from Table 4.3.2.2 and documentation of average strength shall be in accordance with requirements of Section 4.3.3.

TABLE 4.3.2.2 – REQUIRED AVERAGE COMPRESSIVE STRENGTH WHEN DATA ARE NOT AVAILABLE TO ESTABLISH A STANDARD DEVIATION

Specified compressive strength, f'_c, psi	Required average compressive strength, f'_{cr}, psi
Less than 3000 psi	f'_c + 1000
3000 to 5000	f'_c + 1200
Over 5000	f'_c + 1400

4.3.3 – Documentation of average strength

Documentation that proposed concrete proportions will produce an average compressive strength equal to or greater than required average compressive strength (Section 4.3.2) may consist of a field strength test record, several strength test records, or trial mixtures.

4.3.3.1 – When test records are used to demonstrate that proposed concrete proportions will produce the required average strength f'_{cr} (Section 4.3.2), such records shall represent materials and conditions similar to those expected. Changes in materials, conditions, and proportions within the test records shall not have been more restricted than those for proposed work. For the purpose of documenting average strength potential, test records consisting of less than 30 but not less than 10 consecutive tests may be used provided test records encompass a period of time not less than 45 days. Required concrete proportions may be established by interpolation between the strengths and proportions of two or more test records each of which meets other requirements of this section.

4.3.3.2 – When an acceptable record of field test results is not available, concrete proportions may be established based on trial mixtures meeting the following restrictions.

(a) Combination of materials shall be those for proposed work.

(b) Trial mixtures having proportions and consistencies required for proposed work shall be made using at least three different water-cement ratios or cement contents that will produce a range of strengths encompassing the required average strength f'_{cr}.

(c) Trial mixtures shall be designed to produce a slump within ±0.75 in. of maximum permitted, and for air-entrained concrete, within ±0.5 percent of maximum allowable air content.

(d) For each water-cement ratio or cement content, at least three test cylinders for each test age shall be made and cured in accordance with "Method of Making and Curing Concrete Test Specimens in the Laboratory" (ASTM C 192). Cylinders shall be tested at 28 days or at test age designated for determination of f'_c.

(e) From results of cylinder tests a curve shall be plotted showing relationship between water-cement ratio or cement content and compressive strength at designated test age.

(f) Maximum water-cement ratio or minimum cement content for concrete to be used in proposed work shall be that shown by the curve to produce the average strength required by Section 4.3.2, unless a lower water-cement ratio or higher strength is required by Section 4.5.

4.4 – Proportioning by water-cement ratio

4.4.1 – If data required by Section 4.3 are not available, permission may be granted to base concrete proportions on water-cement ratio limits in Table 4.4.

TABLE 4.4 – MAXIMUM PERMISSIBLE WATER-CEMENT RATIOS FOR CONCRETE WHEN STRENGTH DATA FROM FIELD EXPERIENCE OR TRIAL MIXTURES ARE NOT AVAILABLE

Specified compressive strength, f'_c, psi*	Absolute water-cement ratio by weight	
	Non-air-entrained concrete	Air-entrained concrete
2500	0.67	0.54
3000	0.58	0.46
3500	0.51	0.40
4000	0.44	0.35
4500	0.38	†
5000	†	†

*28-day strength. With most materials, water-cement ratios shown will provide average strengths greater than indicated in Section 4.3.2 as being required.

†For strengths above 4500 psi (non-air-entrained concrete) and 4000 psi (air-entrained concrete), concrete proportions shall be established by methods of Section 4.3.

4.4.2 – Table 4.4 shall be used only for concrete to be made with cements meeting strength requirements for Types I, IA, II, IIA, III, IIIA, or V of "Specification for Portland Cement" (ASTM C 150), or Types IS, IS-A, IS(MS), IS-A(MS), I(SM), I(SM)-A, IP, IP-A, I(PM), I(PM)-A, IP(MS), IP-A(MS), or P of "Specification for Blended Hydraulic Cements" (ASTM C 595), and shall not be applied to concrete containing lightweight aggregates or admixtures other than those for entraining air.

4.4.3 – Concrete proportioned by water-cement ratio limits prescribed in Table 4.4 shall also conform to special exposure requirements of Section 4.5 and to compressive strength test criteria of Section 4.7.

4.5 – Special exposure requirements

4.5.1 – Normal weight and lightweight concrete exposed to freezing and thawing or deicer chemicals shall be air entrained with air content indicated in Table 4.5.1. Tolerance on air content as delivered shall be ±1.5 percent. For specified compressive strength f'_c greater than 5000 psi, air content indicated in Table 4.5.1 may be reduced 1 percent.

TABLE 4.5.1 – TOTAL AIR CONTENT FOR FROST-RESISTANT CONCRETE

Nominal maximum aggregate size, in.*	Air content, percent	
	Severe exposure	Moderate exposure
3/8	7-1/2	6
1/2	7	5-1/2
3/4	6	5
1	6	4-1/2
1-1/2	5-1/2	4-1/2
2†	5	4
3†	4-1/2	3-1/2

*See ASTM C 33 for tolerances on oversize for various nominal maximum size designations.

†These air contents apply to total mix, as for the preceding aggregate sizes. When testing these concretes, however, aggregate larger than 1-1/2 in. is removed by handpicking or sieving and air content is determined on the minus 1-1/2 in. fraction of mix. (Tolerance on air content as delivered applies to this value.) Air content of total mix is computed from value determined on the minus 1-1/2 in. fraction.

TABLE 4.5.2 – REQUIREMENTS FOR SPECIAL EXPOSURE CONDITIONS

Exposure condition	Maximum water-cement ratio, normal weight aggregate concrete	Minimum f'_c, lightweight aggregate concrete
Concrete intended to be watertight:		
(a) Concrete exposed to fresh water	0.50	3750
(b) Concrete exposed to brackish water or seawater	0.45	4250
Concrete exposed to freezing and thawing in a moist condition:		
(a) Curbs, gutters, guardrails or thin sections	0.45	4250
(b) Other elements	0.50	3750
(c) In presence of deicing chemicals	0.45	4250
For corrosion protection for reinforced concrete exposed to deicing salts, brackish water, seawater or spray from these sources	0.40*	4750*

*If minimum concrete cover required by Section 7.7 is increased by 0.5 in., water-cement ratio may be increased to 0.45 for normal weight concrete, or f'_c reduced to 4250 psi for lightweight concrete.

4.5.2 – Concrete that is intended to be watertight or concrete that will be subject to freezing and thawing in a moist condition shall conform to requirements of Table 4.5.2.

4.5.3 – Concrete to be exposed to sulfate-containing solutions shall conform to requirements of Table 4.5.3 or be made with a cement that provides sulfate resistance and used in concrete with maximum water-cement ratio or minimum compressive strength from Table 4.5.3.

4.5.3.1 – Calcium chloride as an admixture shall not be used in concrete to be exposed to severe or very severe sulfate-containing solutions, as defined in Table 4.5.3.

TABLE 4.5.3 – REQUIREMENTS FOR CONCRETE EXPOSED TO SULFATE-CONTAINING SOLUTIONS

Sulfate exposure	Water soluble sulfate (SO_4) in soil, percent by weight	Sulfate (SO_4) in water, ppm	Cement type	Normal weight aggregate concrete	Lightweight aggregate concrete
				Maximum water-cement ratio, by weight*	Minimum compressive strength, f'_c, psi*
Negligible	0.00–0.10	0–150	—	—	—
Moderate†	0.10–0.20	150–1500	II, IP(MS), IS(MS)	0.50	3750
Severe	0.20–2.00	1500–10,000	V	0.45	4250
Very severe	Over 2.00	Over 10,000	V plus pozzolan‡	0.45	4250

*A lower water-cement ratio or higher strength may be required for watertightness or for protection against corrosion of embedded items or freezing and thawing (Table 4.5.2).

†Seawater.

‡Pozzolan that has been determined by test or service record to improve sulfate resistance when used in concrete containing Type V cement.

4.5.4 – For corrosion protection, maximum water soluble chloride ion concentrations in hardened concrete at an age of 28 days contributed from the ingredients including water, aggregates, cementitious materials and admixtures shall not exceed limits of Table 4.5.4.

TABLE 4.5.4 – MAXIMUM CHLORIDE ION CONTENT FOR CORROSION PROTECTION

Type of member	Maximum water soluble chloride ion (Cl^-) in concrete, percent by weight of cement
Prestressed concrete	0.06
Reinforced concrete exposed to chloride in service	0.15
Reinforced concrete that will be dry or protected from moisture in service	1.00
Other reinforced concrete construction	0.30

4.5.5 – When reinforced concrete will be exposed to deicing salts, brackish water, seawater, or spray from these sources, requirements of Table 4.5.2 for water-cement ratio or concrete strength and minimum concrete cover requirements of Section 7.7 shall be satisfied.

4.6 – Average strength reduction

As data become available during construction, amount by which value of f'_{cr} must exceed specified value of f'_c may be reduced, provided:

(a) 30 or more test results are available and average of test results exceeds that required by Section 4.3.2.1, using a standard deviation calculated in accordance with Section 4.3.1.1, or

(b) 15 to 29 test results are available and average of test results exceeds that required by Section 4.3.2.1 using a standard deviation calculated in accordance with Section 4.3.1.2, and

(c) special exposure requirements of Section 4.5 are met.

4.7 – Evaluation and acceptance of concrete

4.7.1 – Frequency of testing

4.7.1.1 – Samples for strength tests of each class of concrete placed each day shall be taken not less than once a day, nor less than once for each 150 cu yd of concrete, nor less than once for each 5000 sq ft of surface area for slabs or walls.

4.7.1.2 – On a given project, if total volume of concrete is such that frequency of testing required by Section 4.7.1.1 would provide less than five strength tests for a given class of concrete, tests shall be made from at least five randomly selected batches or from each batch if fewer than five batches are used.

4.7.1.3 – When total quantity of a given class of concrete is less than 50 cu yd, strength tests may be waived by the Building Official, if in his judgment evidence of satisfactory strength is provided.

4.7.1.4 – A strength test shall be the average of the strengths of two cylinders made from the same sample of concrete and tested at 28 days or at test age designated for determination of f'_c.

4.7.2 – Laboratory-cured specimens

4.7.2.1 – Samples for strength tests shall be taken in accordance with "Method of Sampling Freshly Mixed Concrete" (ASTM C 172).

4.7.2.2 – Cylinders for strength tests shall be molded and laboratory-cured in accordance with "Method of Making and Curing Concrete Test Specimens in the Field" (ASTM C 31) and tested in accordance with "Test Method for Compressive Strength of Cylindrical Concrete Specimens" (ASTM C 39).

4.7.2.3 – Strength level of an individual class of concrete shall be considered satisfactory if both of the following requirements are met:

(a) Average of all sets of three consecutive strength tests equal or exceed f'_c.

(b) No individual strength test (average of two cylinders) falls below f'_c by more than 500 psi.

4.7.2.4 – If either of the requirements of Section 4.7.2.3 are not met, steps shall be taken to increase the average of subsequent strength test results. Requirements of Section 4.7.4 shall be observed if requirement of Section 4.7.2.3(b) is not met.

4.7.3 – Field-cured specimens

4.7.3.1 – The Building Official may require strength tests of cylinders cured under field conditions to check adequacy of curing and protection of concrete in the structure.

4.7.3.2 – Field-cured cylinders shall be cured under field conditions in accordance with Section 7.4 of "Method of Making and Curing Concrete Test Specimens in the Field" (ASTM C 31).

4.7.3.3 – Field-cured test cylinders shall be molded at the same time and from the same samples as laboratory-cured test cylinders.

4.7.3.4 – Procedures for protecting and curing concrete shall be improved when strength of field-cured cylinders at test age designated for determination of f'_c is less than 85 percent of that of companion laboratory-cured cylinders. The 85 percent may be waived if field-cured strength exceeds f'_c by more than 500 psi.

4.7.4 – Investigation of low-strength test results

4.7.4.1 – If any strength test (Section 4.7.1.4) of laboratory-cured cylinders falls below specified value of f'_c by more than 500 psi [Section 4.7.2.3(b)] or if tests of field-cured cylinders indicate deficiencies in protection and curing (Section 4.7.3.4), steps shall be taken to assure that load-carrying capacity of the structure is not jeopardized.

4.7.4.2 – If the likelihood of low-strength concrete is confirmed and computations indicate that load-carrying capacity may have been significantly reduced, tests of cores drilled from the area in question may be required in accordance with "Method of Obtaining and Testing Drilled Cores and Sawed Beams of Concrete" (ASTM C 42). In such case, three cores shall be taken for each strength test more than 500 psi below specified value of f'_c.

4.7.4.3 – If concrete in the structure will be dry under service conditions, cores shall be air dried (temperature 60 to 80 F, relative humidity less than 60 percent) for 7 days before test and shall be tested dry. If concrete in the structure will be more than superficially wet under service conditions, cores shall be immersed in water for at least 40 hr and be tested wet.

4.7.4.4 – Concrete in an area represented by core tests shall be considered structurally adequate if the average of three cores is equal to at least 85 percent of f'_c and if no single core is less than 75 percent of f'_c. To check testing accuracy, locations represented by erratic core strengths may be retested.

4.7.4.5 – If criteria of Section 4.7.4.4 are not met, and if structural adequacy remains in doubt, the responsible authority may order load tests as outlined in Chapter 20 for the questionable portion of the structure, or take other appropriate action.